Figure 1: Pore pressure recorded at 9 different locations during one breaching experiment. The ones colored blue and red are the ones shown in the main article.

1 Pore pressure measurements from all 9 locations

We monitor pore pressure at 9 locations (Figure 1). The first 5 (counting from left) of the 9 pore pressure profiles are similar to each other, including the two (colored blue and red) described in detail in the main article. The other 4 sensors recorded the pore pressure drop in response to the initiation, but did not record the pore pressure response to slope failure. This is due to the shortening of the breaching front through time; the location of the other 4 sensors were below the bottom of the breaching front when the front is close by. As a consequence, the horizontal unloading from slope failure did not affect those locations and the sensors do not record any significant pore pressure drop.

At around 100s and 160s, when the blue and red sensor recorded the second pore pressure drop (in response to retrogressive slope failure), the next sensor (red sensor at 100s and green sensor at 160s) seem to have recorded some changes in pore pressure. This may have been caused by the presence of the tubes in the deposit. When the erosion occurs around the tube small slumps are generated sometimes and can cause observable pore pressure change at the next sensor location.

2 Derivation of the steady state pore pressure equation in Lagrangian reference frame

We denote $x$ as the Eulerian reference frame and $\hat{x}$ as the Lagrangian reference frame. In the Lagrangian frame we have $\hat{x} = 0$ at the breaching front. We associate the two
frames with the traveling velocity $v$ of the breaching front as $\dot{x} = x - \int_{t_0}^{t_1} v(t) \, dt$. For a constant velocity, this reduces to $\dot{x} = x - vt$. We use the hat sign for variables referred in the Lagrangian frame. We assume the deposit is homogeneous and the material properties are constant. Consider a representative deposit in the Eulerian reference frame for $x \geq vt$ at time $t$. To conserve fluid mass in this deposit,

$$\frac{\partial n}{\partial t} + (1 - n) \frac{\partial q_f}{\partial x} = 0, \quad x \geq vt \tag{2.1}$$

where $n$ is the volume fraction of the fluid (or porosity), and $q_f (LT^{-1})$ is the flux of fluid. We model the change in porosity as a function of both changes in the mean effective stress ($p' = \frac{\sigma_1 + \sigma_3}{2} (ML^{-1}T^{-2})$) and differential stress ($q = \frac{\sigma_1 - \sigma_3}{2} (ML^{-1}T^{-2})$),

$$\frac{\partial n}{1 - n} = -E \partial p' + \gamma \partial q \tag{2.2}$$

where $E$ is volumetric strain per unit change in mean effective stress and $\gamma$ is volumetric strain per unit change in differential stress; both parameters have unit $ML^{-1}LT^2$ and are positive for the dilative material we are considering in this derivation. Since $p' = p - u$, where $p$ is total mean stress and $u$ is pore pressure:

$$\frac{\partial n}{1 - n} = -E(\partial p - \partial u) + \gamma \partial q \tag{2.3}$$

We assume that the least principal stress is horizontal and the maximum principal stress is vertical. We further assume only the least principal stress changes during breaching, then $\partial p = \frac{\partial \sigma_3}{2}$ and $\partial q = -\frac{\partial \sigma_3}{2}$. Substitute back into Eq. 2.3:

$$\frac{\partial n}{1 - n} = E \left[ \partial u - \left( \frac{1}{2} + \frac{\gamma}{2E} \right) \partial \sigma_3 \right] \tag{2.4}$$

Following Darcy’s law:

$$q_f = -\frac{k}{\mu} \frac{\partial u^*}{\partial x}, \quad x \geq vt \tag{2.5}$$

where $k$ is permeability ($L^2$), $\mu$ is fluid viscosity ($ML^{-1}T^{-1}$), and $u^*$ is excess pore pressure ($ML^{-1}T^{-2}$). Substituting Eq. 2.4 and Eq. 2.5 into Eq. 2.1:

$$\frac{\partial u}{\partial t} - \beta \frac{\partial \sigma_3}{\partial t} - \frac{k}{\mu E} \frac{\partial^2 u^*}{\partial x^2} = 0, \quad x \geq vt \tag{2.6}$$

where $\beta = \frac{1}{2} + \frac{\gamma}{2E}$. We refer to $\beta$ as the relative dilation strength. It represents the ratio between the differential stress induced strain and the mean effective stress induced strain. By this definition, $\beta > 12$ for dilative material. Therefore, in the Lagrangian reference frame:

$$\frac{\partial \hat{u}}{\partial t} - \beta \frac{\partial \hat{\sigma}_3}{\partial t} - v \frac{\partial \hat{u}}{\partial x} + v \beta \frac{\partial \hat{\sigma}_3}{\partial x} - \frac{k}{\mu E} \frac{\partial^2 \hat{u}^*}{\partial x^2} = 0, \quad \hat{x} \geq 0 \tag{2.7}$$

At steady state, $\frac{\partial \hat{u}}{\partial t} = 0$ and $\frac{\partial \hat{\sigma}_3}{\partial t} = 0$. Therefore,

$$-v \frac{\partial \hat{u}}{\partial x} + v \beta \frac{\partial \hat{\sigma}_3}{\partial x} - \frac{k}{\mu E} \frac{\partial^2 \hat{u}^*}{\partial x^2} = 0, \quad \hat{x} \geq 0 \tag{2.8}$$

We assume the water table does not change, therefore changes in excess pore pressure equal changes in total pressure ($\partial u = \partial u^*$) and the steady state pore pressure equation is:

$$\frac{k}{\mu E} \frac{\partial^2 \hat{u}}{\partial x^2} + v \frac{\partial \hat{u}}{\partial x} - v \beta \frac{\partial \hat{\sigma}_3}{\partial x} = 0, \quad \hat{x} \geq 0 \tag{2.9}$$
3 Analytical solution to the steady state equation in Lagrangian reference frame

At steady state in the Lagrangian reference frame, we have

$$\frac{\partial \hat{u}}{\partial t} = 0 \quad (3.1)$$

due to, in the Eulerian frame,

$$\frac{\partial u}{\partial t} + v \frac{\partial u}{\partial x} = 0 \quad . \quad (3.2)$$

Combining with Eq. 2.6 and letting $q_u = \beta \frac{\partial \sigma_3}{\partial t}$ and $C_v = \frac{k}{\mu E}$:

$$C_v \frac{\partial^2 u}{\partial x^2} + v \frac{\partial u}{\partial x} + q_u = 0 \quad (3.3)$$

First solve this equation with $q_u = 0$, i.e.,

$$C_v \frac{\partial^2 u}{\partial x^2} + v \frac{\partial u}{\partial x} = 0 \quad . \quad (3.4)$$

Let $u = g(t)e^{\lambda x}$ (where $g(t) \neq 0$ for non-trivial solution) and substitute into Eq. 3.4:

$$\lambda^2 + \frac{v}{C_v} \lambda = 0 \quad .$$

which gives $\lambda_1 = 0$, and $\lambda_2 = -\frac{v}{C_v}$. Therefore,

$$u = g_1(t)e^{-\frac{v}{C_v}x} + g_2(t)$$

Substitute this into Eq. 3.2,

$$g'_1 e^{-\frac{v}{C_v}x} + g'_2 e^{-\frac{v}{C_v}x} - \frac{v^2}{C_v} g_1 e^{-\frac{v}{C_v}x} = 0$$

$$\Rightarrow g'_1 + g'_2 e^{\frac{v}{C_v}x} = \frac{v^2}{C_v} g_1$$

Since $g_1$ and $g_2$ are only functions of time,

$$g'_1 = \frac{v^2}{C_v} g_1$$

$$g'_2 = 0$$

hence,

$$g_1 = B_1 e^{\frac{v^2}{C_v}t}$$

$$g_2 = B_2$$

where $B_1$ and $B_2$ are constants that can be determined from boundary conditions. Based on the solution to the homogeneous equation (Eq. 3.4), we can assume the solution to the original differential equation (Eq. 3.3) follows

$$u(x,t) = g_1(x,t)e^{-\frac{v}{C_v}x} + g_2(x,t) \quad (3.5)$$
Therefore

$$\frac{\partial u}{\partial x} = \frac{\partial g_1}{\partial x} e^{-\frac{v}{v_C} x} - g_1 \frac{v}{C_v} e^{-\frac{v}{v_C} x} + \frac{\partial g_2}{\partial x} \quad (3.6)$$

Assume that

$$\frac{\partial g_1}{\partial x} e^{-\frac{v}{v_C} x} + \frac{\partial g_2}{\partial x} = 0 \quad (3.7)$$

Substitute Eq. 3.7 into Eq. 3.6:

$$\frac{\partial u}{\partial x} = -g_1 \frac{v}{C_v} e^{-\frac{v}{v_C} x} \quad (3.8)$$

Therefore,

$$\frac{\partial^2 u}{\partial x^2} = -\frac{\partial g_1}{\partial x} \frac{v}{C_v} e^{-\frac{v}{v_C} x} + g_1 \frac{v^2}{C_v^2} e^{-\frac{v}{v_C} x} . \quad (3.9)$$

Substitute equations 3.8 and 3.9 into Eq. 3.3:

$$- \frac{\partial g_1}{\partial x} v e^{-\frac{v}{v_C} x} + g_1 \frac{v^2}{C_v} e^{-\frac{v}{v_C} x} - g_1 \frac{v^2}{C_v} e^{-\frac{v}{v_C} x} + q_u = 0$$

$$\Rightarrow - \frac{\partial g_1}{\partial x} v e^{-\frac{v}{v_C} x} + q_u = 0$$

Combining with Eq. 3.7:

$$\frac{\partial g_1}{\partial x} = \frac{q_u(x, t)}{v} e^{-\frac{v}{v_C} x}$$

$$\frac{\partial g_2}{\partial x} = -\frac{q_u(x, t)}{v}$$

Integrating,

$$g_1 = \frac{1}{v} \int q_u(x, t) e^{\frac{v}{v_C} x} dx \quad (3.10)$$

$$g_2 = -\frac{1}{v} \int q_u(x, t) dx \quad (3.11)$$

For simplicity we write the sink term as

$$q_u(x, t) = \begin{cases} 
-\alpha e^{-\eta(x-vt)}, & x \geq vt, \\
0, & x < vt.
\end{cases} \quad (3.12)$$

where $\alpha$ is magnitude of the source/sink at the boundary ($x = vt$), and $\eta$ is a constant that has a unit of $L^{-1}$. Compare to Eq. (1) and Eq. (2) of the main article, it is easy to see that $\alpha = v\eta/\beta s_0$. Substitute Eq. 3.12 into Eq. 3.10:

$$g_1 = \frac{\alpha}{v} \int -e^{-\eta(x-vt)} e^{\frac{v}{v_C} x} dx$$

$$= -\frac{\alpha e^{\eta t}}{v} \int e^{-\eta x + \frac{v}{v_C} x} dx$$

$$= -\frac{\alpha e^{\eta t}}{v(-\eta + \frac{v}{v_C})} + h_1(t)$$
Substitute Eq. 3.12 into Eq. 3.11:

\[
g_2 = -\frac{\alpha}{v} \int e^{-\eta(x-vt)} \, dx
\]

\[
= \frac{\alpha e^{\eta vt}}{v} \int e^{-\eta x} \, dx
\]

\[
= -\frac{\alpha e^{\eta vt} e^{-\eta x}}{v \eta} + h_2(t)
\]

Therefore,

\[
u = g_1(x,t) e^{-\frac{w}{c_v} \hat{x}} + g_2(x,t)
\]

\[
= -\frac{\alpha e^{\eta vt} e^{-\eta x}}{v(-\eta + \frac{w}{c_v})} + h_1 e^{-\frac{w}{c_v} \hat{x}} - \frac{\alpha e^{\eta vt} e^{-\eta x}}{v \eta} + h_2
\]

\[
= -\frac{\alpha}{v} \left( \frac{C_v}{v - \eta C_v} + \frac{1}{\eta} \right) e^{-\eta (x-vt)} + h_1 e^{-\frac{w}{c_v} \hat{x}} + h_2.
\] (3.13)

We write \( A_0 = \frac{\alpha}{v} \left( \frac{C_v}{v - \eta C_v} + \frac{1}{\eta} \right) \) for convenience in the following derivations. At steady state, \( \frac{\partial u}{\partial t} + v \frac{\partial u}{\partial x} = 0 \). Therefore,

\[
\frac{\partial u}{\partial t} = -v \frac{\partial u}{\partial x}
\] (3.14)

Substitute Eq. 3.13 into Eq. 3.14,

\[
-v \eta A_0 e^{-\eta (x-vt)} + h_1' e^{-\frac{w}{c_v} \hat{x}} + h_2' = -v \eta A_0 e^{-\eta (x-vt)} + \frac{v^2}{C_v} h_1 e^{-\frac{w}{c_v} \hat{x}}
\]

\[
\Rightarrow h_1 + h_2' e^{-\frac{w}{c_v} \hat{x}} = \frac{v^2}{C_v} h_1
\]

Hence,

\[
h_1' = \frac{v^2}{C_v} h_1 \Rightarrow h_1 = A_1 e^{\frac{v^2}{C_v} t}
\]

\[
h_2' = 0 \Rightarrow h_2 = A_2
\]

where \( A_1 \) and \( A_2 \) are constants. As a result,

\[
u = A_1 e^{-\frac{w}{c_v} (x-vt)} - A_0 e^{-\eta (x-vt)} + A_2
\] (3.15)

or in the Lagrangian frame where \( \hat{x} = x - vt \),

\[
\hat{u} = A_1 e^{-\frac{w}{c_v} \hat{x}} - A_0 e^{-\eta \hat{x}} + A_2.
\] (3.16)

Or, in the excess pore pressure form,

\[
u^* = A_1 e^{-\frac{w}{c_v} (x-vt)} - A_0 e^{-\eta (x-vt)} + A_2'
\] (3.17)

and in the Lagrangian frame,

\[
\hat{u}^* = A_1 e^{-\frac{w}{c_v} \hat{x}} - A_0 e^{-\eta \hat{x}} + A_2'
\] (3.18)
where $A'_2 = A_2 - u_h$, $u_h$ is the hydrostatic pore pressure at a given point. Boundary conditions are

$$
\begin{align*}
\hat{u}^*(0, t) &= 0 \\
\hat{u}^*(\infty, t) &= 0
\end{align*}
$$

Therefore, $A'_2 = 0$ and $A_1 = A_0 = \frac{2}{\tau} \left( \frac{C_v}{v - \eta C_v} + \frac{1}{\eta} \right) = \frac{\alpha}{\eta C_v}$. For our specific problem, $\alpha = v \eta \beta s_0$, where $\beta$ is the relative dilation strength and $s_0$ is the far field horizontal stress. Then

$$
\hat{u}^* = \frac{v \beta s_0}{v - \eta C_v} \left( e^{-\frac{v}{\tau} \hat{x}_m} - e^{-\eta \hat{x}} \right)
$$

(3.19)

We have from the main article (Eq. (3))

$$
v = -\frac{k}{\mu E_0 \hat{u}_{\min}} \frac{\partial \hat{u}}{\partial \hat{x}} \bigg|_{\hat{x}=0}
$$

(3.20)

where $E_0$ is a constant that has a unit of $M^{-1}LT^2$. We solve for $\hat{u}_{\min}$ and its corresponding $\hat{x}_{\min}$ from Eq. 3.19,

$$
\frac{\partial \hat{u}^*}{\partial \hat{x}} \bigg|_{\hat{x}_{\min}} = 0
$$

$$
\Rightarrow \quad \frac{v \beta s_0}{v - \eta C_v} \left( -\frac{v}{C_v} e^{-\frac{v}{\tau} \hat{x}_{\min}} + \eta e^{-\eta \hat{x}_{\min}} \right) = 0
$$

$$
\Rightarrow \quad \eta e^{-\eta \hat{x}_{\min}} = \frac{v}{C_v} e^{-\frac{v}{\tau} \hat{x}_{\min}}
$$

$$
\Rightarrow \quad \hat{x}_{\min} = \frac{C_v}{\eta C_v - v} \log \left( \frac{\eta C_v}{v} \right)
$$

Substitute this into eq 3.19 and notice $\eta e^{-\eta \hat{x}_{\min}} = \frac{v}{C_v} e^{-\frac{v}{\tau} \hat{x}_{\min}}$,

$$
\hat{u}^*_m = \frac{v \beta s_0}{v - \eta C_v} \left( e^{-\frac{v}{\tau} \hat{x}_{\min}} - e^{-\eta \hat{x}_{\min}} \right)
$$

$$
= \frac{v \beta s_0}{v - \eta C_v} \left( e^{-\frac{v}{\tau} \hat{x}_{\min}} - \frac{v}{\eta C_v} e^{-\frac{v}{\tau} \hat{x}_{\min}} \right)
$$

$$
= \frac{v \beta s_0}{v - \eta C_v} \left( 1 - \frac{v}{\eta C_v} \right) e^{-\frac{v}{\tau} \hat{x}_{\min}}
$$

$$
= -\frac{v \beta s_0}{\eta C_v} e^{-\frac{v}{\tau} \hat{x}_{\min}} \log \left( \frac{\eta C_v}{v} \right)
$$

$$
= -\frac{v \beta s_0}{\eta C_v} \left( \frac{\eta C_v}{v} \right) \frac{v}{v - \eta C_v}
$$

(3.21)

Substitute the solution for $\hat{u}_{\min}$ into Eq. 3.20:

$$
v = \frac{k}{\mu} \frac{\eta C_v}{v \beta s_0} \left( \frac{\eta C_v}{v} \right) \frac{v \beta s_0}{C_v}
$$

$$
\Rightarrow \quad 1 = \frac{k}{\mu} \frac{C_v v}{v \beta s_0} \left( \frac{\eta C_v}{v} \right) \frac{v \beta s_0}{C_v}
$$

$$
\Rightarrow \quad 1 = \frac{E}{E_0} \left( \frac{\eta C_v}{v} \right) \frac{v \beta s_0}{C_v}
$$

(3.22)
To solve Eq. 3.22 and 3.19, we introduce $\delta$ here defined as $\frac{v}{\eta C_v}$. Substitute this definition into Eq. 3.22 we can solve for $\delta$ as follows:

$$\Rightarrow 1 = \frac{E}{E_0} \left( \frac{1}{\delta} \right) \frac{1}{\delta}$$

$$\Rightarrow \delta = \frac{E}{E_0}$$

$$\Rightarrow \delta = \frac{W\left[ \frac{E}{E_0} \log\left( \frac{E}{E_0} \right) \right]}{\log\left( \frac{E}{E_0} \right)}$$

(3.23)

where $W(x)$ is the Lambert W-function. To ensure that $v > 0$, we need $E_0 > E$. Finally, the steady state solutions for pore pressure and erosion rate are

$$\dot{u}^* = \frac{\delta \beta s_0}{1 - \delta} \left( e^{-\eta \dot{x}} - e^{-\frac{v}{\eta C_v} \dot{x}} \right)$$

(3.24)

$$v = \delta \eta C_v$$

(3.25)

And substitute the solutions into Eq. 3.21 we have

$$u_{\text{min}}^* = \frac{\beta s_0 E}{E_0}.$$  

(3.26)

4 Testing of material for mechanical properties

We run flow-through experiments on the sample to measure its permeability. In this experiment a sand sample is prepared in an cylindrical acrylic tube with a diameter of about 5cm. The inner wall of the tube is coated with the same material as the sample. The head drop over the sample is controlled by the difference in elevation of two water reservoirs that are pumped to the two ends of the sample. This water head difference drives water flow through the sample. We record the water flux ($q_f$) through the sample and the corresponding head drop $\Delta h$. The permeability is calculated as,

$$k = \frac{q_f L}{\Delta h A}$$

(4.1)

where $L$ is the total length of the sample, and $A$ is the cross sectional area of the sample. We run the experiment on both the silty sand and the well-sorted fine sand used in breaching experiment. We find on average, the permeability for the silty sand is $4 \times 10^{-14} m^2$ and the permeability for the well-sorted fine sand is $4 \times 10^{-12} m^2$.

We use the triaxial cell to measure the isotropic unloading compressibility and the relative dilation strength. We isotropically unloading the sample by gradually decrease the cell pressure $u_c$ while keeping the pore pressure $u_p$ constant. We calculate the isotropic unloading compressibility as the slope between the volumetric strain $\varepsilon_v$ and the mean effective stress $p' = u_c - u_p$ of the sample.

$$E = \frac{d\varepsilon_v}{dp'}$$

(4.2)

The measured $E$ for the silty sand is $3.5 \times 10^{-7} Pa^{-1}$, and that for the well-sorted fine sand is $4.3 \times 10^{-7} Pa^{-1}$. 
We run drained shear test to measure the relative dilation strength of the material. During the test both the cell pressure and the pore pressure are held constant and their difference is small to maintain low confining stress levels (from 3kPa to 21kPa) on the sample. The sample is free to drain and we monitor the change in pore volume through the pump connected to the sample. We shear the sample with a constant axial (vertical) strain rate of about 10%/hr. At the end of each test we calculate the volumetric strain ($\varepsilon_v$) and the corresponding differential stress $q$ at different time. Then we calculate the relative dilation strength as follows,

$$
\beta = 1 + \frac{1}{2E} \frac{d\varepsilon_v}{dq}
$$

where $E$ is the isotropic unloading compressibility. The relative dilation strength started to be around 1 and increases with continuous shearing; $\beta = 4.5$ near failure.

## 5 Estimation of modeling parameters $\eta$ and $s_0$

We use the initial pore pressure response to fit the two parameters. Because the largest pore pressure drop occurs very close to the normal pore pressure boundary (the breaching front) and the permeability of the silty sand is large we can not consider the initial response as undrained. Instead we use a numerical model to simulate the pore pressure response to the initiation.

In this model we use equation 2.7 with $v = 0$ and $\frac{\partial \hat{\sigma}_3}{\partial t} = ae^{-\eta x}$, where $a$ is the rate of stress change at the breaching front. This stress distribution model is consistent with what we use for the breaching model (Equation 2 of the main article). We assume that the stress change at the breaching front was at a constant rate during the lifting of the restraining plate. We set this time to be 0.5s based on observation of the experiments, then $a = 2s_0$. The total simulation time is 0.5s. We find the spatial distribution of the initial pore pressure response is only sensitive to $\eta$ and the magnitude of the initial pore pressure response depends on $s_0$. Therefore we can use the recorded initial pore pressure distribution and magnitude to constrain those two parameters.

## 6 Breaching condition

From stability analysis based on Mohr-Coulomb failure criterion [2], we find

$$
\frac{\sigma'_1}{\sigma'_3} = \frac{\sigma_1 - u_c}{\sigma_3 - u_c} = \frac{1 + \sin(\theta)}{1 - \sin(\theta)}
$$

where $\theta$ is the internal friction angle and we assume it is 30°, and $u_c$ is the critical pore pressure that can just ensure stability of the deposit. Solve this equation we get:

$$
u_c = -\frac{(1 - \sin(\theta))\sigma_1 - (1 + \sin(\theta))\sigma_3}{2\sin(\theta)}
$$

We define the critical excess pore pressure to be $u^*_c = u_c - \rho_f gh$, where $\rho_f$ is the density of the fluid and $h$ is the depth to the point of interest. For simplicity, we let $\sigma_1 = \rho_b gh$
and $\sigma_3 = \rho_f gh$. This simulate the stress condition at the breaching front where there is no horizontal force except the force from water pressure. Substitute them into the above equation and denote $\rho_{bb} = \rho_b - \rho_f$ as the buoyant bulk density, we have:

$$u_c^* = \frac{\sin(\theta) - 1}{2 \sin(\theta)} \rho_{bb} gh = -\frac{\rho_{bb} gh}{2} \tag{6.3}$$

When the magnitude of the actual excess pore pressure $u^*$ is larger than that of the critical excess pore pressure, i.e., $\frac{u^*}{u_c^*} > 1$, the deposit is stable, otherwise slide of the deposit occurs. We use the minimum pore pressure of the steady state solution ($u_{min}$) to substitute the actual pore pressure $u^*$ in the criterion, thus

$$\frac{u_{min}^*}{u_c^*} > 1 \Rightarrow \frac{\beta s_0 E}{E_0} \frac{2}{\rho_{bb} gh} > 1$$

$$\Rightarrow \beta > \frac{E_0 \rho_{bb} gh}{2 s_0 E} \tag{6.4}$$

We assume initially the horizontal stress equals half the buoyant vertical loading, $s_0 = \rho_{bb} gh/2$, therefore we can reduce the above equation to

$$\beta > \frac{E_0}{E} \tag{6.4}$$

We call Eq. 6.4 the breaching condition. It defines a threshold for the relative dilation strength $\beta = \frac{E_0}{E}$. Once $\beta$ is larger than this value, the ratio between the minimum excess pore pressure and the critical excess pore pressure is larger than 1, ensures stability of the deposit.

7 Estimate the relative dilation strength of silty sand from Scripps Canyon from published simple shear test results

Dill [3] published results from simple shear tests on the sediment samples they collected from the head of the Scripps Canyon. The test results includes the normal stress ($\sigma_N$) and shear stress ($\tau$) during shearing and the volume change after shearing. We can calculate the relative dilation strength $\beta$ based on those data. First, we estimate the mean and differential stress of the sample during shear. We assume the measured stresses are at critical state where the stress Mohr circle of the sample is tangential to the failure envelope (Figure 2). The geometric relationships suggests

$$q = \frac{\tau}{\cos \phi} \tag{7.1}$$

$$p = \sigma_n + \tau \tan \phi \tag{7.2}$$

where $\phi$ is the internal friction angle of the deposit (assumed to be 30°). Since the test was done in a drained condition, i.e., no abnormal pore pressure, the mean effective stress ($p'$) is the same as the mean stress ($p$). By definition,

$$\varepsilon_v = -Ep' + \gamma q \tag{7.3}$$
Figure 2: Mohr circle of the testing sample at critical state where the shear stress does not change but the sample can continue to deform. Measured normal stress ($\sigma_N$) is OB, measured shear stress ($\tau$) is BC, mean stress ($p$) is OA, and the differential stress ($q$) is AC. Line OC is the failure envelope and the angle AOC is the internal friction angle $\phi$.

where the volumetric strain $\varepsilon_v$ is calculated from the initial volume ($v_0$) and the change of volume after shear ($\Delta v$) as $\varepsilon_v = \Delta v/v_0$. Both the compressibility $E$ and the parameter $\gamma$ are unknown. Due to lack of additional test data, we assume a typical compressibility value for the sample ($10^{-8} Pa^{-1}$) and calculate the value of $\gamma$. Then we can calculate the dilation strength $\beta$ with

$$\beta = \frac{1}{2} + \frac{\gamma}{2E}$$  \hspace{1cm} (7.4)

References

